SOME MECHANICS PROBLEMS IN EARTHQUAKE ENGINEERING

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Since the beginning of earthquake engineering research in the United States in the 1920's, this discipline has proved to be a particularly fruitful source of interesting problems in applied mechanics. Some examples of the earliest such problems are the development of the response spectrum as a tool in analysis and design, the development of nonlinear hysteretic models of structural response for dynamic loading, and the application of the theory of stochastic processes to problems in modeling of strong ground motion and structural response. An additional class of problems has arisen from efforts to understand the effects of soil-structure interaction on structural response. The need for dynamic analyses in order to understand and simulate earthquake response has also been one of the major factors behind the development of modern computer codes for structural analysis.

Earthquake engineering continues to be a source of many challenging problems in dynamics and other fields of mechanics and I have made a selection of some of these problems for discussion here. The problems have been chosen to give some idea of the variety of problems that exist, and also to show problems at various stages of understanding. The selection of problems also reflects, of course, my own interest and experience.

STRUCTURAL DYNAMICS

The Appendage Problem

As students looking for a Ph.D. topic will attest, most structural dynamics problems in earthquake engineering seem to be either complicated or already solved. There is one fairly straight forward problem in linear structural dynamics, however, that has resisted solution until very recently. This is the equipment or appendage problem and the solution to which I am referring is that by Sackman and Kelly (1978). The problem is illustrated in Fig. 1 and consists of an N degree-of-freedom structure to which a small oscillator, the equipment, is added. The most interesting case occurs when the equipment mass, m, is small, and the fixed-base natural frequency of the equipment is near or coincident to one of the natural frequencies of the structure. In this case one recognizes the problem as that of the dynamic vibration absorber with the significant difference that the application is to transient motion, rather than steady-state response. The simplest case which has the essential features of the problem is the two-degree-of-greedom system shown in Fig. 2.

Consider the case where k/m and K/M are equal, and the damping is small or zero. In addition, $m \ll M$. This is the tuned case, in the parlance of the problem. Such a structure, when considered as a two degree-of-freedom oscillator, will have two natural frequencies, one slightly above $\sqrt{K/M}$ and the other slightly below. As indicated in Fig. 2, in the fundamental mode shape both masses will deflect in the same direction, while in the second mode they will deflect in opposite directions. Under earthquake excitation, beginning from rest, these two modes will begin the response in phase. Gradually, as the earthquake goes on, the two modes will become out of phase, the displacement of the large mass will decrease and that of the small mass will increase, and the relative displacement between the two will grow. Still later, the modes will be in phase again, and the relative displacement will become quite small. Thus, if one concentrates on the relative motion, or absolute acceleration, of the top mass, a beat phenomenon is seen. This is illustrated in Fig. 3, which shows the absolute acceleration of the upper mass. Depending on the parameters of the system, the beat period can be quite long, and the earthquake can be over before the response has built up to its maximum. In this case the maximum occurs in the free vibrations which follow the earthquake, and can be significantly smaller than the peak response to longer motions of the same strength.

The reader is referred to the publication of Sackman and Kelly for the details of the solution, including formulas for the application of response spectrum techniques to the problem.

In calculating the response for design studies, the tempting approach, particularly when the equipment mass is very small, is to assume that the interaction force is negligible, and to calculate the response of the structure, without equipment, at the point of attachment. This response is then used as input to the equipment. This approach, which is often acceptable in the "detuned" case, has the practical advantage that the input need not be recalculated if changes are made in the design or selection of the equipment. As the above discussion has tried to point out, however, the interaction forces can be quite large in the tuned case: even though the mass is small, the deflection can grow large enough to generate a significant force. Another way to see this is to realize that the tuned equipment provides a harmonic force at a frequency very near one of the resonant frequencies of the structure. It therefore acts like a vibration generator shaking the building at resonance. If the damping of the structure is small, the effects on the total structural response can be very important. For example, by hand exciting a large fan-motor assembly that was coincidentally tuned to the fundamental frequency of the building, two of us were able to excite 9-story reinforced concrete building to levels where ~~when~~ the motion could be felt ~~flet~~ by the occupants of the upper floors.

As I mentioned in the beginning, investigators in earthquake engineering have been aware of this problem for some time, and although some features of it have been illuminated, for example, by Ruzicka at Illinois, the major credit for its successful solution belongs to Sackman and Kelly.

Systems Identification

The next mechanics problem in earthquake engineering that I want to discuss has the rather ungrammatical title of systems identification. As applied to earthquake engineering problems, this reduces typically to the problem of identifying the parameters of a structural model from its response, usually with the aid of the excitation. Occasionally, only non-parametric descriptions of the structure are sought, such as given by transfer functions or impulse response functions. There is much current research activity in this area, including a session at this conference, and I am not going to attempt to review the field. Instead, I will simply try to give some insight into some recent work two of our students, James Beck (1978) and Graeme McVerry (1979), have done at Caltech.

Assume for simplicity that we are interested in determining the parameters of a symmetric structure responding in a single direction.

The problem is planar and it is assumed that both the earthquake excitation and the response at one or more levels are known. For linear structures, the problem can be approached either in the time or frequency domains, the approaches are fundamentally equivalent. For nonlinearly responding structures, the frequency domain approach is limited, e.g., to the determination of time-varying, equivalent linear parameters, and more general examinations must be made in the time domain.

For this discussion, we will limit ourselves to linear models and will assume further that the masses are known. Fig. 4 illustrates the problem in both the time and frequency domains. The first question that arises is: What is identifiable and what is not? For the type of systems which occur in earthquake engineering, for example, tall framed structures, it turns out that limitations in the number of recording instruments and other practical considerations require identification of the parameters of the dominant modes of response. These are the period, damping and participation factor times the mode shape ordinate at the recording level. Identification of the individual elements of the stiffness and damping matrices is rarely a practical possibility. In general, all the elements of [K] and [C] can be determined uniquely only if the response is measured at all degrees of freedom. If one is willing to settle for a finite number of possibilities for [K] and [C], from which a single pair can be selected by sufficient other information l.c. (This is termed local identifiability), then Beck has shown that response at half or more of the degrees of freedom is still needed.

Identifiability of the system improves if the model is a chain system. The structure shown in Fig. 4 is such a system if it is assumed that the columns are inextensible. In this case, (again with [M] known), Udwadia, Sharma, and Shah, (1978) have shown that the system is locally identifiable if the input and response at one location are known. The elements of [K] and [C] can be found uniquely if the response is known at the first level above the base. This interesting, almost paradoxical, result is of very limited practical importance in earthquake engineering, however, because the implied calculations are inherently ill-conditioned: the determination of the stiffness and damping elements requires knowledge of the relative displacements of the systems at asymptotically high frequencies, well beyond the highest natural frequency of the structure. Such information is not normally available from earthquake records; in the typical case the earthquake response is dominated by the lower modes of vibration and high frequency excitation and response are below the level of the noise in the records.

Fig. 5 is a schematic illustration of one of the simplest approaches to systems identification in the time domain. In this approach, both the mathematical model and the real structure are subjected to the measured excitation and the calculated and measured responses are compared. The measured response and input are the basic data, of course, and the calculated response is considered to be a function of the parameters of the modes considered. If the model were capable of representing the structure, if the correct values of the parameters were taken, and if only the selected number of modes participated in the actual response, then the calculated and measured responses would coincide. This never happens, naturally, and the parameters of the modes are determined to minimize the square of the difference between the two responses. Because of the large number of parameters that can be involved, it pays in application to earthquake response data to take advantage of the linearity of the least-squares minimization with respect to some of the parameters. An application of the method developed by Beck is shown in Figs. 6 through 9, and in Table I. Figs. 6 and 7 show the building studied, which is a 9-story steel-framed structure at the Jet Propulsion Laboratory (JPL) in Pasadena. The building was strongly shaken in the San Fernando earthquake of 1971, experiencing a base acceleration of 0.2g, and a roof response of about 0.4g. The results of the analysis are summarized in Table I, and the degree of fit obtained between the calculated and measured acceleration and velocity are illustrated in Figs. 8 and 9, respectively. The degree of fit possible by the methods of systems identification is extremely good by earthquake engineering standards and is significantly better than has been achieved by the trial-and-error approaches previously applied to the problem. This degree of fit allows the damping, particularly that in higher modes, to be identified with significantly better accuracy than has heretofore been the case. Natural periods of vibration are also found more accurately and more periods are occasionally identified, but their accuracy is not significantly greater than that from other methods, except when the periods vary during the response. In this case systems identification allows the changes to be found more accurately than other approaches.

It is seen also in Table I that some of the values for the damping are considered questionable: they seem too high. Perhaps the most important reason for this difficulty is that the highest mode considered in the analysis is more sensitive than the lower modes to the effect of the still higher modes not included, i.e., when a three-mode model is used, the third mode is, in a sense, forced by the method to take parameters that account for the combined effects of the third and higher modes.

Identification of structural properties from earthquake response has previously been done in the frequency domain, primarily by examination of the moduli of experimentally determined transfer functions, such as that shown in Fig. 10. The figure is typical of transfer functions obtained from earthquake response, and it should not be surprising that obtaining information beyond the first two or three natural periods has proven to be difficult. To assist the interpretation is has been common practice to smooth such functions by averaging. The physical significance of such averaging is not altogether clear for the earthquake problem, but the averaging does reduce and broaden all peaks, which degrades the data. Another major difficulty with the typical transfer function approach is that the effects of the truncation of the records tend to be ignored. Further problems are occasioned by any nonlinearity or time variation in the structure itself. This often happens in response to strong shaking and can, for example, broaden resonant peaks in transfer functions as a result of loss of structural stiffness and consequent lengthening of natural periods. All of these problems have combined to severely limit the use of typical transfer function approaches in earthquake engineering.

McVerry's contribution, which represents a significant advance in this type of approach, is comprised of several parts. First, he concentrated on the unsmoothed, complex-valued transfer function, rather than the smoothed modulus. Second, he developed the terms that are required when finite transforms such as the FFT are used on a segment of excitation and

and response; thus terms specifying the difference in the state of the system at the beginning and end of the time segment comprise part of the parameters that are identified. Finally, he considered the functional forms implied by the selected structural model from the beginning of the analysis, rather than bringing the model in at the final step.

With these points in mind, the method can be briefly described as the choosing the structural parameters of the model to provide a least squares fit of the calculated and observed, finite Fourier transforms of a selected duration of excitation and response. The frequency band over which the minimization is performed is usually as broad as low and high frequency noise will permit, consistent with the model. When systems identification by the transfer function approach is done this way, the comparison of the calculated and observed responses are equally as good as results obtained in the time domain.

An example is illustrated in Figs. 11 through 13. Fig. 11 is Caltech's Millikan Library, which also was shaken hard by the San Fernando earthquake. It is a nine-story reinforced concrete structure. The lateral resistance in the N-S direction is provided by external shear walls while that in the E-W direction is provided primarily by shear walls in the core. Results of McVerry's analysis for a two-mode model of the structure in the N-S direction are shown in Figs. 12 and 13. Results in Fig. 12 are for a time-invariant system, while those in Fig. 13 are for time windows of various durations as indicated. It is seen in Fig. 12 that very good agreement is obtained in both the time and frequency domains. The agreement in the frequency domain is shown by the ~~agreement~~ match between the moduli of the calculated and observed acceleration transforms of roof response. One of the practical results of this example is seen in Fig. 13, where the changes of the modal parameters with time are plotted. It is hard to see trends in values for the second mode, although the levels are defined, but clear trends in first mode period and damping are seen. The first mode damping is seen to start from a low value, increase quickly to about 8 percent as the response builds toward a maximum. By 12 seconds it has dropped to 5 percent, where it remains for the rest of the significant response.

These results and others in this field show that it is possible to extract more information from earthquake response records than has previously been thought possible. In particular, accurate values of the lengthening of the periods of the buildings and effective damping values of the modes can only be obtained by these techniques at the percent time. The next important step, which must await data from future earthquakes, is the identification of structural properties of buildings responding in the highly nonlinear range. I should mention at this point that McNiven and his co-workers at Berkeley have successfully applied similar techniques to the nonlinear problem using data from shaking table tests of small structures. Their latest results are to be presented at this conference.

Tipping

There are many interesting problems in mechanics that have been drawn from the problems of dynamic soil-structure interaction, which is the term used to describe problems of the dynamics of structures founded on a flexible media. Depending on circumstances, the response of such structures can be significantly different from those on rigid bases. In the setting of this conference, I should point out that a sizeable fraction of the better work in this field has been done by Anestis Veletsos at Rice and Jose Roesset at the University of Texas, Austin.

One interesting problem in this field is that of a structure that can tip, i.e., soil-structure interaction with lift-off. The problem is shown schematically in Fig. 14 which illustrates incipient tipping of a building and two mechanical idealizations of the problem. It is assumed in the figure that the bottom of the structure is restrained against horizontal motion. Barring liquefaction of the soil, no buildings have tipp~~ing~~ed completely over during earthquakes; the application is more subtle. What seems possible is that under extremely strong shaking, some relatively rigid buildings may begin to tip. In such a case, even a small amount of uplift may be enough to dissipate significant energy and to reduce markedly the fundamental frequency of the structure, thereby reducing its acceleration response and internal forces. And, of course, small objects do fall over completely in earthquakes, and an understanding of their dynamics would be useful.

First, one should realize that the ability to tip does not require flexibility of the structure nor of the ground. The amplitude-dependent frequency and effective damping of a rigid block rocking on a rigid plane has been solved by Housner (1963). In this case the natural period of the structure (block) changes from zero to a finite value when rocking occurs, and the damping arises from the impacts of rocking, wherein the vertical momentum of the center mass is changed and energy is lost.

The simplest model of a structure founded on a flexible foundation would arise if the foundation were replaced by two springs, one near each corner of the structure as shown in Fig. 15. Such a model embodies some of the essential features of the problem, but is not capable of dissipating energy. A dashpot in parallel with the spring and a dashpot with limited travel are possible additions to the system to allow the dissipation of energy (see the third part of Fig. 14). The fact that a rigid block rocking on a rigid surface dissipates energy, while one on springs does not, and one on an elastic half space dissipates energy by radiation only, suggests that there are subtleties to this problem. For example, a good question for an oral Ph.D. exam is to ask, for the horizontally restrained block on two springs initially at rest, whether the center of the mass starts to move up or down after lift-off, if the block is given a horizontal impulse sufficient to break contact with one of the springs.

Again consider the block on two springs restrained against horizontal motions at the base (Fig. 15). If lift-off does not occur, the system has two degrees-of-freedom and by inspection the modes are a purely vertical motion, and rocking about the center of the base. If the structure, when rocking only, moves enough to lift-off, it finds itself supported by only one spring. In this state the structure is also linear with two modes. The modes have coupled rocking and vertical motion, but if the springs are relatively stiff, the typical case for buildings and soils, one mode will be comprised primarily of vertical motion with a high natural frequency and one will be primarily rocking, with a low natural frequency. When the structure reestablishes contact with the disengaged spring, it resumes its original modes and frequencies. Thus the problem is an alternation of two linear problems, and the state of the system at the end of each linear episode determines the initial conditions for the two modes of the next linear segment of response. With dashpots added that give damping believed appropriate for rocking of buildings, ~~and~~ the resulting motions appear~~s~~, intuitively at least, capable of describing what might occur in stiff, low-rise buildings. For the earthquake response of spring-mounted equipment, or in other applications, the induced vertical motion might be much more important.

Another somewhat more realistic idealization is the Winkler foundation shown in the center of Fig. 14. In this case the equations of motion after lift-off are complicated considerably by the variable length of contact between the block and the foundation. As a next step in complexity, it is also possible to replace the block by a flexible structure such as a shear beam or a frame with separate footings. Again the equations of motion are complicated, but tractable on the digital computer.

The analysis of tipping has not progressed as far as the previous two problems, and several interesting features of the problem await future work. For example, continued experimental research and the development of design techniques along the lines of the work of Priestley et al. (1978) are needed.

DETERMINATION OF LOCAL MAGNITUDE

The next problem I want to discuss is from strong-motion seismology and concerns the use of engineering instruments to determine the magnitude of earthquakes. The nature of the application of mechanics is quite different from the previous problems. As originally defined by C. F. Richter, the local magnitude, M_L, of an earthquake is determined by the logarithm to the base 10 of the response in millimeters of a Wood-Anderson seismograph located 100 km from the epicenter. A scaling constant is introduced so a magnitude 3 earthquake at this distance produces a record of 1 millimeter, and the effects of different recording distances are accounted for by emperical curves of the attenuation of Wood-Anderson response with epicentral distance. The Wood-Anderson seismograph has a natural period of 0.8 sec, a damping factor of 0.8 and a gain of 2800. In practice, instruments with different dynamic properties are often used to determine M_L by taking into account their amplification of ground motions in the frequency range near 0.8 sec.

The magnitude is important in earthquake engineering because of its central role in determining earthquake resistant design criteria. In the design of major projects such as dams and power stations, the design earthquakes are nearly always specified in terms of their magnitudes and a large body of research exists relating the character of ground motion to earthquake magnitude. In this application, the local magnitude, M_L, and the surface wave magnitude, M_S, are the most commonly used scales.

Professor Hiroo Kanamori at Caltech's seismological laboratory and I have been working on this topic, but from the reverse viewpoint; we have been using the records of strong-motion instruments, accelerographs and seismoscopes, to determine the local magnitude of earthquakes (Kanamori and Jennings, 1978; Jennings and Kanamori, 1979). It is the second of these studies that I wish to discuss here.

Consider the seismoscope shown in Fig. 16. It is essentially a conical pendulum. Typically, it has a natural period near 0.75 second and a damping value near 0.10. The two-dimensional response of the instrument is scribed by a stylus on a smoked watch glass. A representative record is shown in Fig. 17. The instrument is low in cost and requires almost no maintenance.

It does not, of course, record as much information as a strong-motion accelerograph, but it does provide directly a representative point on the response spectrum of the ground motion.

Because the local magnitude is determined by the maximum response of an instrument with a nearly equal period, it occurred to us that the seismoscope could be used to determine M_L if a correction could be made for the different dampings and the slight change in period. Correcting for the different gains of the two types of instruments requires only a simply-determined multiplicative factor. If the corrections for period and damping could be found, it would expand the instrumental base for determining M_L, as the seismoscope is much less sensitive than the Wood-Anderson instrument, and has only gone off scale a few times under very intense motion. The standard Wood-Anderson instrument goes off scale under excitation that is on the threshhold of human perceptibility.*

The instrumental correction factor was found by an unusual application of a well-known result of random vibration theory (Crandall and Mark, 1963). If a one degree-of-freedom oscillator with unit mass is subjected to a force which is a white noise with mean zero and spectral density D, then after stationarity is achieved, the mean of the response is zero and the mean square is given by

$$\langle x^2 \rangle = \frac{DT^3}{16\pi^2\zeta} \qquad (1)$$

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*Special Wood-Anderson seismographs with a gain of 4, 1/700 the standard sensitivity, have a dynamic range that can record accelerations up to about 20 percent g before going off scale, but there are very few instruments of this type in use.

in which T is the period and ζ the damping of the oscillator. The result also describes the mean square, steady-state response of an ensemble of oscillations with these properties subjected to an ensemble of white noise excitations with spectral density D. In addition, it has been established that if the probability distribution of the imput to the oscillator is Gaussian, so is its response.

The conditions of this result are met approximately in earthquake response if the excitation is broad-band, which it usually is, and if the duration of strong shaking is long with respect to the period T. In this application T is near 0.8 sec and this last condition is also usually satisfied. Also, the amplitudes of earthquake accelerations are well-described by a Gaussian probability distribution.

Thus the response can be taken as Gaussian with zero mean and, statistically, the amplitude of the response is determined by the single parameter of the mean square, i.e., the amplitude of the response of different oscillators to the same input, including the peak response, scales approximately as $(T^3/\zeta)^{\frac{1}{2}}$. Letting V stand for the instrumental gain, and subscripts wa and sc denote the Wood-Anderson seismograph and the seismoscope, respectively, the desired formula is

$$A_{wa} = \frac{V_{wa}}{V_{sc}} \sqrt{\left(\frac{T_{wa}}{T_{sc}}\right)^3 \left(\frac{\zeta_{sc}}{\zeta_{wa}}\right)} \; A_{sc} \qquad (2)$$

Fig. 18 shows how this statistically-derived formula works in individual cases and is based on sites where both accelerograph and seismoscope data are available. The plot shows the Wood-Anderson response extrapolated from the seismoscope records using equation 2, and, for the same instrument site, the Wood-Anderson response calculated on the digital computer by using the measured accelerogram as input to the equations of motion of the seismograph. The data are from three different earthquakes, primarily San Fernando, 1971 and Parkfield, 1966, and include all components of U.S. data where both seismoscope and accelerograph data are available. If the correction formula were exact, all points in Fig. 18 would fall on the straight line. This is not expected, of course, but the relatively small scatter indicates a surprisingly good result. Nearly all the points are within ±40 percent of the line, which indicates differences in M_L of the order of only 0.2. Most points are much closer than 40 percent, indicating the error in M_L introduced by the statistically-derived correction is quite small. This is borne out, for example, by data from 16 sites in the San Fernando earthquake in which we found $M_L = 6.34 \pm 0.19$ from accelerograph data and $M_L = 6.44 \pm 0.20$ from seismoscope response. The two values are within one-half of a standard deviation of each other, and have about the same dispersion. A similar agreement was found for the data from the Parkfield earthquake, although only four joint sites were available. It seems clear that the error introduced in the statistical extrapolation is acceptably small, lying within the scatter caused by the source mechanism of the earthquake, the radiation pattern, geological heterogeneity, etc.

Thus, this very simple idea seems to work very well.

THE CAPACITY OF STRUCTURES

The next problem I wish to discuss is much more complex than the foregoing. It is, however, one of the central structural dynamics problems in earthquake engineering. The problem is the determination of the ultimate capacity of a given structure to resist earthquake motions. The problem is illustrated by the response of North Hall of the University of California, Santa Barbara, to the earthquake of August 13, 1978 (Miller and Felszeghy, 1978, Porter, et al., 1979). As seen in Figs. 19 through 22, the building is a rectangular (34 x 240 ft) three-story building of shear wall construction. There was an error in the earthquake-resistant design of the building which resulted in the lateral loads for design being 1/10 of their intended value. This was discovered prior to the earthquake and several additional shear walls of reinforced concrete were added (Fig. 22) to bring the structures up to the standards of the 1976 Uniform Building Code. The building was subjected to forced vibration tests before and after the strengthening, so its linear dynamic properties are known (Hart, et al., 1978). After strengthening, the building was instrumented to record earthquake response under the program of the State of California's Office of Strong-Motion Studies. The response of the building to the $M_L = 6.0$ earthquake is given in Figs. 23 through 25. The building was located approximately 14 km from the center of the fault plane of the earthquake. It is seen from Fig. 23 that the maximum horizontal ground accelerations were 0.41g and 0.38g, with a much smaller vertical ground acceleration, 0.11g. The response near the top of the structure is given by traces 5, 7, 8, and 9 in the N-S direction, and trace 6 in the longitudinal, E-W direction. (Figs. 24 and 25)

The three N-S records on the third floor show peak accelerations of 0.58g (No. 8), 0.68g (No. 5) and 0.63g (No. 9), moving from west to east. The roof response in the same direction is 1.03~~5~~9. From the records it appears the largest peaks in the response occurred in the fundamental mode, which shows a period of about 0.3 sec. Depending on the mode shape and the mass distribution, these records indicate a value of base shear of from 50 to 70 percent of the weight of the structure. In the longitudinal E-W direction, the maximum acceleration of the third floor was about 0.56g, indicating response slightly less severe than in the transverse direction. The damage to the building consisted of light-to-moderate X-cracking of the new shear walls. The cracking occurred over the height of the shear walls, with the heaviest cracking in the N-S walls at the first story. Very little cracking occurred in the older shear walls. These were constructed of concrete block, as seen in Fig. 19, and are more flexible than the new reinforced concrete walls. The damage was not hazardous, is easily repairable, and the loss of structural strength and integrity because of the earthquake is minimal. The structure obviously could have successfully resisted much stronger excitation, or the same level of excitation for a much longer time.

This is a success story for earthquake engineering: a potentially hazardous building was identified and strengthened, and subsequently survived very high levels of earthquake motion without serious damage. It should be clear, however, that the situation is not necessarily a victory for engineering mechanics. The provisions of the building code are such as to require in this case that the structure be able to resist at yield level a base shear on the order of 20 percent of the weight of the structure, and yet the building received approximately three times this amount during the earthquake, and it is seen from the damage that the response exceeded the yield level by only a small amount. The very difficult questions for emchanics are: "What is the capacity of the structure against collapse?" and "Can the conservatism resulting from standard codes and practices be explained and documented from basic principles?" These questions are not as practically relevant when dealing with the design of traditional buildings as they are in other applications. For the traditional buildings, the gradual evolution of the codes and the emperical evidence of structural response basically control the situation. However, in the design of critical structures such as dams, nuclear power plants and storage facilities for liquified natural gas, the design process is examined very critically, and where strengths and capacities cannot be clearly established, conservative steps are taken. The result is often a very expensive structure, which engineers "know" is overconservative--but they are unable to "prove" it.

In the case of unusual buildings, the problem can be the reverse. It is possible to eliminate the traditional sources of extra strength in the structure by special architectural and engineering features. If such a structure is designed to resist only the forces of the code, rather than the actual forces expected during strong shaking, a potentially hazardous structure can result.

The challenge to engineering mechanics posed by these problems is that of accurately predicting the capacity of structures under extreme dynamic loading. Progress has been made, of course, and some of the reasons for the observed discrepancies between design loads and capacities are known to various degrees. These include the conservative relation between actual and specified properties of materials, the conservative specification of sizes of structural elements, and the neglect of beneficial or detrimental "non-structural" elements. Quantitatively, however, most of the work in determining the capacity of structures remains to be done.

ROCK SLIDES

The last problem in mechanics arising from earthquake engineering that I wish to discuss is that of rock falls which turn into flow slides. This is a problem ~~about~~ in which very little is known about the mechanics, and which has intrigued me for a long time. In this country, the most recent examples were generated by the Alaskan earthquake of March 27, 1964. The best known of these massive flows was the Sherman Glacier slide shown in Figs. 26 and 27, but other large slides also occurred as illustrated in Figs. 28 and 29 (Post, 1967). A prehistoric slide of this type, the Blackhawk slide, also occurred on the north slope of the San Bernardino mountains of Southern California. These slides can be extremely hazardous; a similar slide in the Peruvian earthquake of 1970 killed 25,000 to 30,000 people when Yungay and other villages were buried (Cluff, 1971). One of the most interesting and potentially most hazardous features of these slides is their ability, once generated, to flow large distances over almost flat slopes. This can be seen from the photographs, and from the charts in Fig. 30. The mechanism of these slides has been studied by R. L. Shreve (1968), who has put forward a hypothesis of air cushioning to explain the distance traveled, the evidence of skipping, and other features of the slides. Such a mechanism seems possible when slides such as shown in

Figs. 26 and 28 are seen, but seem less likely to be the explanation for the slide in Fig. 29. In addition, similar appearing slides have been observed in the Capri Chasm on Mars, where air cushioning cannot operate.

It seems to me that at least some of the slides, therefore, admit an alternate hypothesis. The fine, striated structure of the slides, the tendrils seen in some cases, the large average size of the particles and the features mentioned previously suggest to me the following possibility: The rock mass breaks loose, dropping several hundreds of feet or more virtually as a unit. During this part of the slide the mass breaks up into pieces, but mostly along pre-existing joints and fractures. Very little energy is lost to air resistance. When the gentler slope is reached, if the mass has sufficient kinetic energy (some slides, like Hebgen Lake, 1959 stop essentially as a plug at the bottom of the fall), the rock mass moves out like a fluid. Because of the angularity and size of the particles, and the lack of an effective pore fluid, the flow has a rather thick "boundary layer" with the rocks on the bottom layer rolling on the contact surface, the next layer rolling on them, etc. The process is highly frictional, and although the rocks start this part with a high kinetic energy, it is dissipated during the flow, and at the final stage, the mass "locks up" or "freezes" from the bottom upward and possibly backward from the front. Such a locking process is possible in a highly frictional flow and might explain the detailed structure seen in some of the slides.

To investigate this speculative hypothesis further, I had a student make analyses of rolling layers of bodies governed by the elementary theory of friction. This model does admit locking when things slow down enough, and requires that large particles roll further than small ones, which are *two features* of the problem. Other than this, and the gaining of some additional insights, we did not get very far, and I have no real idea whether my hypothesis is correct. The problem at this stage is still, I believe, very far from explanation. I might add that this is a problem that requires good experimental work, and one of the objectives of the study mentioned above was to give some insight into the type of experiment that might duplicate the phenomenon in a laboratory setting.

CLOSURE

The problems discussed above are all drawn from important problems in earthquake engineering and illustrate the range of problems in mechanics that can be generated by the engineering problems of a practical discipline. In return, the careful application of the principles of mechanics to these problems has contributed much to the extant of practical solutions that now exist.

The problems were presented in approximate order of decreasing level of understanding and range from one whose central feature is solved, to one in which, I believe, the mechanicsm is not understood even qualitatively. There are many other problems in earthquake engineering that could equally well have been presented. These include problems in dynamics testing of structures of all types, in fluid-structure and soil-structure interaction, in mathematical modeling of structures, in the development of finite element methods for structural analysis, and a wide variety of problems in the dynamics of soils.

As a final comment, I would like to reiterate the importance of the interplay between mechanics and in this case, the practical problems of earthquake engineering. The interaction is beneficial for both sides: without mechanics, earthquake engineering would proceed slowly and dangerously via empericism; and without the stimulus of earthquake engineering, many important problems in mechanics would be overlooked.

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LIST OF CAPTIONS

Figure No.	Caption
1	Schematic representation of the appendage probelm: a small, single degree-of-freedom oscillator attached to a large, multi-degree-of-freedom structure, all subjected to earthquake motion. (Sackman and Kelly, 1978)
2	The two degree-of-freedom appendage problems. The figure on the left shows the structure and appendage; the figures on the right show the mode shapes and frequencies.
3	Acceleration of the appendage during earthquake response, showing the beat phenomenon that can occur. (Sackman and Kelly, 1978)
4	The equivalence of the parameters of linear structural models in the time and frequency domains. The masses are assumed known in most applications of systems identification in earthquake engineering.
5	simple Illustration of a single case of the application of systems identification to the earthquake response of a building.
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Figure No.	Caption
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19	North Hall on the campus of the University of California, Santa Barbara (UCSB). (Miller and Felszeghy, 1978)

Figure No.	Caption
20	Plan and elevation of North Hall, UCSB, showing locations of accelerometers installed by the Office of Strong-Motion Studies, California Division of Mines and Geology. Heavy lines on the plan view denote shear walls. (Porter et al., 1979)
21	Third floor and roof plans of North Hall, UCSB, showing accelerometer locations. See Fig. 20. (Porter et al., 1979)
22	Floor plan of North Hall, UCSB, showing locations of original (concrete block) and added (reinforced concrete) shear walls. The new walls were added after an error was found in the calculations for the seismic design. (Miller and Felszeghy, 1978)
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24	Acceleration records from accelerometers 4, 5 and 6 at North Hall, UCSB, during the Santa Barbara earthquake of August 13, 1978. See Figs. 20 and 21 for instrument locations (Porter et al., 1979)
25	Acceleration records from accelerometers 7, 8 and 9 at North Hall, UCSB, during the Santa Barbara earthquake of August 13, 1978. See Figs. 20 and 21 for instrument locations. The roof record, trace 7, has a peak over one g and is the largest response so far recorded in this country. (Porter et al., 1979)
26	Sherman glacier rockslide, which occurred during the Alaska earthquake of March 27, 1964. The source of the rock is the peak in the top center of the photograph. Photograph taken August 25, 1965. (Post, 1967)
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Figure No.	Captions
29	Allen glacier rockslide No. 1, Alaska earthquake of March 27, 1964 as seen on August 25, 1965. Note the overlapping digitate structure. (Post, 1967)
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TABLE I

Optimal Estimates of the Parameters of the Longitudinal Modes of JPL Building 180. Date Used are the First 20 Seconds of the S82°E Records.

Modal Parameter[1]	1-Mode Model (Displacement match)	1-Mode Model (Velocity match)	1-Mode Model (Acceleration match)	2-Mode Model (Velocity match)	2-Mode Model (Acceleration match)	3-Mode Model (Acceleration match)
T_1	1.27	1.26	1.26	1.26	1.25	1.25
ζ_1	2.0	2.6	3.5	2.5	4.2	4.2
p_1	0.9	1.1	1.3	1.0	1.5	1.5
T_2				0.35	0.37	0.38
ζ_2				14(?)	13(?)	5.3
p_2				-0.52	-0.47	-0.24
T_3						0.3
ζ_3						12 (?)
p_3						-0.4
$J^{1/2}$(%)	15.1	14.2	12.6	12.2	8.2	6.9

[1] T = period; ζ = damping factor; p = effective participation factor; J = error normalized by the product of the length of record and the maximum observed response.